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BRAC Ill KC-10

MAINTENANCE HANGAR COMPLEX

McGUIRE AIR FORCE BASE

NEW JERSEY

5 August 1994

FOUNDAMoN ENGINEERING e SOIL AND ROCK NECHANICS 9 CONSTRUC-nON CONSULTATION

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5 August 1994

Frankfurt Short Bruza
Architects, Engineers, Planners
5701 North Shartel, Suite 210
Oklahoma City, OK 73118

Attn: Mr. Arthur E. Austin, Jr.

Re: BRAC 111 KC-10 Maintenance Hangar Complex
McGuire Air Force Base, New Jersey

Gentlemen:

This report contains the results of our geotechnical investigation as well as the foundation des-Lgn recommendations for the proposed maintenance hangar, fire fighting system, and pavements located near Building 1807 at McGuire Air Force Base in New Jersey. A separate report dated 23 June 1994 addressed the foundation conditions at the proposed control tower.

SUBSURFACE INVESTIGATION

The subsurface investigation for the maintenance hangar was performed by Warren Ceorge, Inc., between May 25 and June 17, 1994 under our engineering supervision. The investigation consisted of the drilling of thirty-one borings varying between ten and fifty feet in depth.

Continuous sampling was performed in the top ten feet and at five foot intervals from there on. All sampling was done in accordance with ASTM D-1586. The site investigation included electric resistivity testing, where practical, as well as testing for the corrosivity of the soil. Drawing 1 shows the boring location plan for the maintenance hangar. Logs of the individual borings are included in this report.

SUBSURFACE CONDITIONS

McGuire Air Force Base is located on the outer costal plane province of Burlington County, New Jersey. The local strata are tertiary formations which include Kirkwood sand and Hornerstown marl. The Kirkwood sand is a miocene deposit consisting of stratified deposits of fine micaceous sands and silt with lesser amounts of*clay and gravel. The Kirkwood outcrops dip gently increasing in thickness to the southeast. Underlying the Kirkwood sands is the Hornerstown marl, an eocene deposit. The Hornerstown marl is a green glauconitic marl with varying amounts of sand, silt, and clay. During the drilling operations some of the borings penetrated up to three feet into a green fine sand with silt, presumed to be the Hornerstown marl. The regional bedrock is reported at depths greater than 100 feet.

Drawings 2 to 4 show profiles through the soil at the maintenance hangar. Surface elevations at the site vary from +117 to +121. A one-foot-thick concrete slab covers most of the site, except for five borings 'in the grassy apron to the south and west. The underlying layer of natural soil, designated as Stratum 1, is a grey and orange-brown layer of fine sand, with little silt and occasional fine gravel. There are occasional layers and pockets of coarse to fine sand with little fine gravel. Stratum 1 varies from seven to fifteen feet in thickness, and is medium dense with occasional loose pockets. A few of the borings show a three to five foot thick medium dense layer of grey silt with varying'amounts of fine sand, designated as Stratum la. The next major stratum, designated as Stratum 2*,, is well graded grey coarse to fine sand with trace silt and occasional pockets of medium to fine gravel. The layer is ten to fifteen feet thick, and is medium dense to dense. Underneath Stratum 2, one of the borings showed a pocket of peat. Some of the deeper borings terminate in Stratum 3, a medium dense to dense layer of silt and fine sand or fine sand and silt. Five of the deepest borings penetrate into a dark green sand or silt, designated as Stratum 4. This stratum is believed to be the glauconitic Hornerstown marl, which is very dense according to the May 1951 investigation for the existing hangar. The soil types encountered are not expected to be susceptible to consolidation, swelling or heaving, and therefore should cause minimal differential settlement.

The groundwater table was encountered between Elevations +107 and +109 during the soil investigation. The data are consistent with the ground water tables measured between Elevations +106 and +110 in the 1951 borings. The concrete pavement is sloped to provide drainage of the surface into several storm sewers. During construction the groundwater table may fluctuate since the concrete pavement will no longer shield the ground from infiltration of rain, or evaporation.

DESCRIPTION OF FACILITIES

The proposed aircraft hangar will have three bays for aircraft servicing, as well as a warehouse. The proposed floor slab is at Elevation +120. Typical single column service loads are understood to range from 50 to 150 kips compression, with no uplift. Some of the X-braced columns in the hangar area have transient service loads as high as 750 kips compression, between 400 and 500 kips uplift, and 250 kips horizontal shear.

The fire fighting system includes two above ground fire water tanks, forty feet in diameter, fifty feet tall, located fifty feet to the north of the warehouse area. The tanks will be built on a continuous concrete ringwall, with the top of the ringwall at Elevation +121. In addition there is a 75 by 89 foot underground concrete AFFF containment tank f6r fire fighting foam. The tank has two-foot-thick roof'and floor slabs, and an approximately fourteen-foot-high reservoir chamber. The top of tank Elevation is +121; it is located forty feet west of the maintenance hangar.

The pavement areas includes an aircraft approach ramp currently designed as a 14-inch concrete slab, over a six inch layer of concrete stabilized base-course, over eight inches of free draining gravel with an under-drainage system.

FOUNDATION RECOMMENDATIONS

Spread Footings

Columns that experience no net uplift forces can be placed on spread footings designed for a maximum bearing pressure of 4000 psf. The bearing pressure may be increased by one third for wind load combinations. The ultimate lateral load on these spread footings should be calculated as fifty-five percent of the total vertical load acting on the footing, including the weight of the soil and the concrete, and an appropriate factor of safety should be applied. A soil unit weight of 120 pcf, and a concrete unit weight of 150 pcf should be used in t-he calculation of the overburden load. No footing dimension should be less than three feet.

The highly loaded X-braced columns in the maintenance hangar are subject to vertical compression, uplift, as well as shear. If spread footings are used to suppoit these columns, they should be designed with appropriate factors of safety for all possible load combinations. For vertical compression the same bearing pressure as for columns experiencing no uplift applies. The uplift resistance of the spread footings for'X-braced columns should be taken as the weight of the concrete and soil directly above the bottom of the spread footing, plus shear along the sides. An appropriate factor of safety should be applied to the weight in computing the uplift capacity. The side shear resistance should be calculated as a linearly increasing load of 10 psf per foot of depth from zero at the top of'the soil layer to the bottom of the footing, and can be*applied along the perimeter of the footing. The side shear value is a service resistance, and includes a factor of safety of two.

The horizontal shearing force may be resisted by base friction and passive resistance. The ultimate base friction should be fifty-five percent of the compression load for the appropriate load combination, including all structural as well as overburden loads, over the area of the footing. Footings may be tied together with grade beams below the floor slab level to transfer the horizontal shearing force to as many footings as needed. If additional resistance to horizontal loads is required, it may be necessary to utilize the friction on the bottom of the floor slab. The ultimate base friction should be calculated as 55 percent of the slab weight. An appropriate factor of safety applied should be applied in the calculation of the service capacity to horizontal loads.

Passive resistance should only be relied upon if base friction cannot develop sufficient resistance. The passive pressure should be calculated as an equivalent fluid load increasing by 50 psf per foot of depth from zero at the ground surface to the bottom of the footing, along the side of the footing that will bear on the soil. The passive resistance is based on a displacement compatible with that needed to mobilize friction on the footing bases. If passive resistance is used to resist lateral loads, the footing excavation must be backfilled and compacted under full time inspection to ensure proper compaction.

Some of the interior partition walls consist of 12-inchthick masonry walls, intended to be supported on 16-inchwide grade beams. The grade beams must be designed for compatible settlement with the adjacent spread footings to prevent cracking. Near the spread footings the shear may cause a gross upward pressure of up to 8000 psf on the bottom of the grade beams. The grade beams would therefore require reinforcement for shear and reverse bending of 8000 psf minus the dead load of the CMU wall. For design purposes, this loading may be assumed to act over a distance of five feet from the footing. As an alternative, the grade b-eams may be designed to frame all structural loads into the spread footings by leaving a small void or some very loose soil underneath the grade beam. CMU walls on properly reinforced or thickened slabs should only be build after the adjacent spread footings have undergone all settlement due to dead load.

Pile Foundations

In the case of the X-braced columns the design is likely to be governed by uplift and horizontal shearing forces. For these columns it may be more cost-effective to install pile foundations. The pile caps or adjacent spread footings should be tied together with grade beams to transfer the horizontal shearing forces to as many foundations as needed. Additional capacity may be realizad by mobilizing the base shearing resistance of the floor slab, as discussed above. Batter piles are not considered to be a cost effective or practical method of resisting lateral loads for this project.

Design alternatives considered included H-piles and concrete filled steel pipe piles. Other common pile types were deemed inappropriate for these soil and loading conditions. A wave equation analysis of pipe piles showed that it may be difficult to achieve the full sixty foot embedment once the pile is driven into the underlying dense layer presumed to be the Hornerstoin, marl. The alternative of more pipe piles driven Lo &, lesser embedment would be too costly. H-piles should achieve the full sixty-foot penetration, due to the-'-r heavier cross section and their non-displacement characteristic.

Piles are presumed to have a minimum spacing of thre~, pile diameters. The vertical pile capacities given below are based on analyses of individual piles, as well as group action, and include a minimum factor of safety of two. The lateral pile capacities for resisting horizontal forces are based on the presumption of fifty percent fixity at the top of the piles, with displacements compatible with that needed to mobilize friction on footing bases and floor slabs, and need no additional factor of safety. H-piles are assumed to be positioned for strong axis bending.-

We recommend the HP 14 x 73 as the most practical and cost effective pile for the project. The pile has a design capacity of 75 kips in compression, 50 kips uplift, and a lateral resistance of 7 kips per pile for the first row of piles, and 3 kips per pile in subsequent rows. The design bending moment for the lateral loadA is 15.5 kip-feet. The minimum penetration resistance using a Vulcan 08 hammer is 30 blows per foot.

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Alternate pile types and capacities include:

- HP 12 x 53 H-Pile: 60 kips in compression, 35 kips uplift, and a lateral resistance of 6 kips per pile for the first row of piles, and 2.5 kips per pile in subsequent rows. The design bending moment for the lateral load is 12.1 kip-feet.

- HP 10 x 42 H-Pile: 50 kips in compression, 25 kips uplift, and a lateral resistance of 4 kips per pile for the first row of piles, and 2 kips per pile in subsequent rows. The design bending moment for the lateral load is 7.2 kip-feet.

All piles should be driven in single sixty foot lengths. Any splices made in the field must develop the full tensile strength of the pile. The piles should be driven to a minimum embedment of sixty feet, with the minimum penetration resistance specified above. We anticipate that penetration into the dense layer will require much harder driving, with a resistance of up to 150 blows per foot for the HP 14 x 73. We recommend a minimum of two load tests in uplift, and two load tests in compression for each type of pile used.

To achieve adequate fixity of the piles in the pilecap-, the heads of the piles should be fixed in the pilecap by straps welded to the pile. A minimum of four straps to be welded to the H-pile flanges should be designed for the full uplift force and bending moment listed abcve.

GeneraJ Design Recommendations

The design depth for frost penetration has economic implications on the cost of the project, particularly the pavement design. The commentary to the Unified Building Code and NAVFACS DM-7.1 give contour maps that indicate a depth of frost penetration between thirty and forty inches. Local construction practice uses a value of forty-eight inches. Measurements during last winters extreme conditions showed frozen ground to twenty-four inches. We recommend a value of forty inches.

The lateral earth pressure for excavations at the maintenance hangar may be calculated as an equivalent fluid load increasing 40 psf per foot of depth to the water table, and then increasing at 75 psf per foot of depth.

FIREWATER STORAGE TANKS

The firewater tanks are expected to have a water load of up to 3000 psf. Once structural and live load are added, the bearing, pressure on the tanks **should not exceed 4000** psf. The total settlement of the tanks is expected to be between 4 and 5 inches at the tank center, and between 2 and 3 inches at the edge. The influence of the adjacent tank may increase the settlement at the edge between 1.5 and 2 inches, and between 1 and 1.5 inches at the center of the adjacent tank. All pipe connections should be designed to accommodate the appropriate movements.

AFFF STORAGE TANK

The AFFF tank should be designed so that it will nct become buoyant. The design water table for this typ,:,- of structure should be the ground surface, a condition consistent with extreme flooding during a possible storm. The resistance to buoyancy should be provided by the weight of the concrete structure, and the weight of any soil on top of the roof, or on top of spread footings under the tank, plus a minimal safety factor. Resistance from shearing along the sides may not be relied upon. Design alternatives may include tension piles to hold the tank in place during a flood, or a fail-safe system designed to let the tank flood uniformly in case of the water table reaching a certain elevation.

The AFFF tank walls should be designed for lateral loads with an adequate factor of safety for typical groundwatet conditions, but should still have a minimal factor of safety on their ultimate strength in case the groundwater level should ever rise to the ground surface.

The excavation support for the AFFF tank needs to be properly designed for the lateral soil pressures given in this report. The permeability of the second soil stratum encountered during the investigation is expected to be very high, with a large radius of influence, and large scale pumping from deep wells may cause adverse settlements in adjacent buildings or pavements. Prescribing the method of construction is likely to impact costs, and we therefore recommend that the detailed construction and dewatering plan be developed by a qualified engineer retained by the Contractor and submitted for review and approval.-

PAVEMENT,DESIGN

The California Bearing Ratio (CBR) for the pavement design may be taken as 10 based on soil type, which corresponds to a k-value of 200 pci. A major concern for the design of pavements is the frost susceptibility of the upper soil stratum of fine sand with little silt. The capillary rise in this layer is high enough for water to reach the frozen ground and cause frost heave. We believe this to be one of the contributing factors for the pavement failures observed at the site. Any material placed as subbase for the pavement should be separated from the underlying frost susceptible soil by an appropriately selected geotextile, such a Mirafi 140 NS or 140 NSL drainage fabric. Likewise a layer of geotextile should be placed between the gravel and the concretestabilized sub-course to prevent fines and concrete fragments from clogging the gravel.

A positive, but very expensive method of preventing frost heave is the removal and replacement of all frost susceptible soils to the depth of frost penetration. Alternatively, a geotextile enclosed drainage barrier installed underneath the pavement would cut off capillary water from the frost susceptible material above. Finally, the CBR value could be reduced for design purposes to account for possible softening of the subbase due to frost action.

ELECTRIC RESISTIVITY

Electric resistivity tests were conducted with a Megger Null Balance Earth Tester (Cat. No. 250241) using the three pin fall-of-potential method. In this configuration the three electrodes are spaced at the desired distance, and the resistivity is measured at sixty-two percent of the separation. The depth of the investigation is equal to the distance between the two outer electrodes. The test results are presented in Table 1.

The paved surface and many utility locations at both the control tower and the maintenance hangar made testing at most boring locations impractical. A total of three locations were tested. One was fifty feet south-west of the control tower, the other two were in the grassy apron next to borings MH-17 and MH-30. The measured resistivities are very high, and are indicative of the dry soils encountered close to the surface.

CORROSIVITY

A total of four soil samples were tested for their pH and corrosivity. The corrosivity is evaluated using the Langelier Index, a method that uses the pH, total alkalinity, calcium hardness, total dissolved solids and temperature to evaluate whether the water will dissolve or deposit calcium carbonate. The attached letter by Associated Analytical Laboratories, dated July 13, show that the conditions at the control tower are moderately aggressive, while the conditions at the maintenance hangar vary from nonaggressive to highly aggressive. We recommend that Type II cement be used in the construction of all footings, pile caps, grade beams, floor slabs, and any other concrete structures in direct contact with the soil.

We trust that the information contained in this report is sufficient for the design of the maintenance hangar facilities.

Yours very truly,

Tonis Raamot

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